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Design of a Steel Highway Bridge

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DESIGN OF A
STEEL HIGHWAY BRIDGE

BY

WALDEMAR KARKOW

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

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College of Engineering

May 24, 1913.

I recommend that the thesis prepared under my supervision by WALDEMAR KARKOW entitled Design of a Steel Highway Bridge be approved as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

F. O. Dufour

Asst. Professor of Structural Eng'g.

Recommendation approved

Ira O. Baker

Head of Department of Civil Eng'g.

247402

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I. INTRODUCTION.

This thesis has been prepared to fulfill part of the requirements for the degree of Bachelor of Science in Civil Engineering. The work taken up will include the investigation of the highway bridge located on the State Road over Salt Fork Creek, about one and one-half miles south of St. Joseph, Illinois, and the design of a new bridge to replace the old one. As the average graduate from engineering colleges has only hazy ideas as to cost this thesis will take up the total cost of the undertaking and in that manner endeavor to eliminate or decrease this lack of knowledge.

II. OLD BRIDGE.

Art. I. Specifications.

The specifications to be used in the investigation of the old bridge are Ostrup's Specifications for Highway Bridges, edition of 1911.

Art. 2. Description.

This bridge, built in 1867 by Z. King and Co. of Cleveland, is a wrought iron, class C., bridge of the parabolic bowstring type; and consists of one one-hundred-foot span and one eighty-four-foot span. A view of the bridge is shown in Plate I., while the details of the one-hundred-foot span are shown in Plate II.

This bridge is in bad condition. The vibrations due to passing wagons are very marked; the chord braces afford the upper chord very little support while some of the floor beams are so split or twisted out of shape that ~~they~~ in many places do not support the stringers. Throughout the whole bridge connections are made by bending the members into the desired shape. All field rivets are hand-driven.

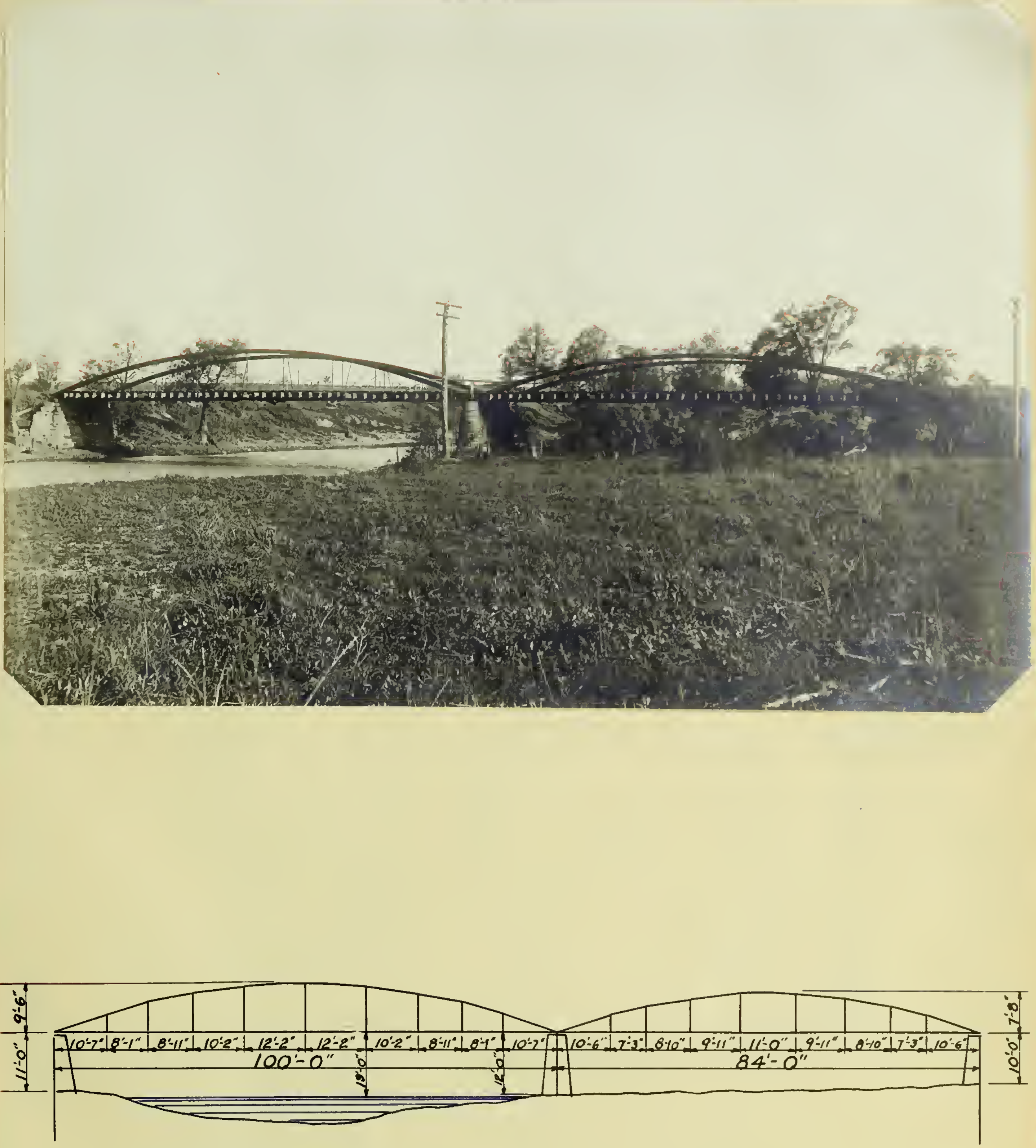


PLATE I VIEW and GENERAL DIMENSIONS

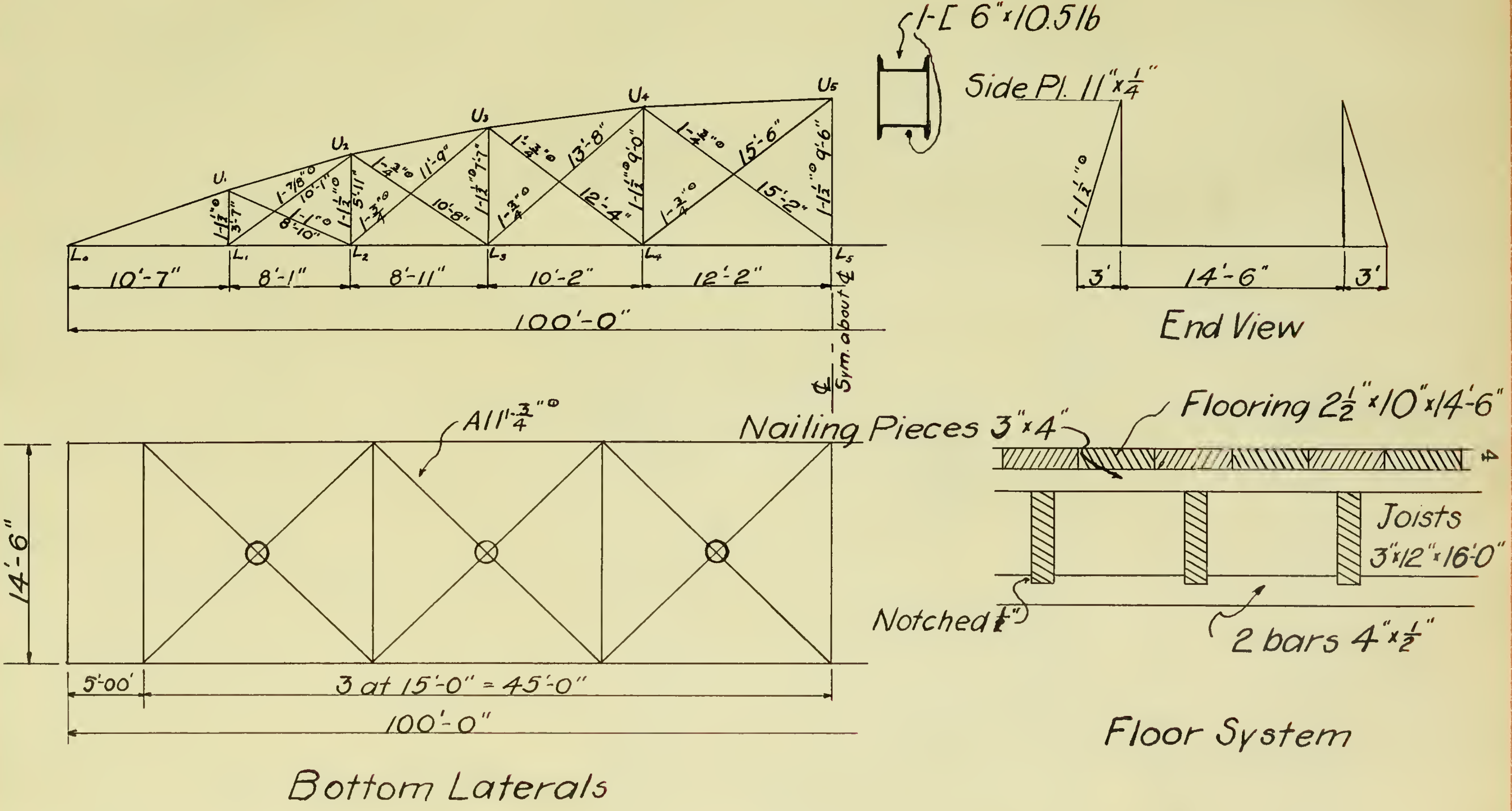


PLATE II DETAILS of 100-FOOT SPAN

Art. 3. Calculation of Stresses.

As the sections of the short span are nearly equal to those of the long span, only the long span will be investigated as it will be the weaker of the two.

The computation of the dead load is put in tabular form and is shown in Table I.

The live load is taken at eighty pounds per square foot of entire roadway.

The stresses are calculated by the graphical method. This solution and the stress diagram are shown in Plate III. In the stress diagram the stresses are recorded in the following order: dead load, +live load, -live load, impact, maximum, and minimum.

Table I.

Weights of one hundred-foot Span.

No.	Shape	Size	Length Feet.	In.	Wt. per ft.	Total Weight.	
			Two Top Chords.				
4	[	6" 10½#	102'	0"	10.5	4,285.0	
4	side pls.	11" x ¼"	102'	0"	9.35	3,820.0	
20	splice pls.	11" x ¼"		6"	9.35	93.5	
4,160	riv. hds	5/8"			$\frac{9.95}{100}$	414.0	
							8,612.5
			Two Bottom Chords				
4	bars	4" x ½"	100	0	6.80	2,720.0	
20	splice pls	4" x ½"		11	6.80	124.8	
240	riv.hds	5/8"			$\frac{9.95}{100}$	23.9	
							2,868.7

No.	Shape	Size	Length Feet.	In.	Wt. per ft.	Total Weight.	
			Verticals.				
4	rods	1½"	5	1			
4	"	1½"	7	5			
4	"	1½"	9	1			
4	"	1½"	10	6			
4	"	1½	11	0			
4	"	1½	43	1	6.01	1064.0	
40	nuts	3"			$\frac{317.5}{100}$	127.0	
40	washers				$\frac{40}{100}$	16.0	1207.0
			Diagonals				
4	rods	1-1/8"	6	2	3.38	83.3	
4	"	1"	9	4	2.67	76.2	
4	"	1/8"	10	7	2.04	86.5	246.0
4	"	¾"	11	2			
4	"		12	3			
4	"		13	3			
4	"		14	2			
4	"		15	8			
4	"		16	0			
4	rods	¾"	82	6	1.15	494.0	
40	nuts	1½"			41.0	16.4	
18	castings				25.0	450.0	960.4

No.	Shape	Size	Length Feet.	In.	Wt. per ft.	Total Weight.	
			Top Chord Bracing.				
4	Rods	1½"	8	0	6.01	192.2	
2	Rods	1½"	10	0	6.01	120.2	
3	[s	6" 10½#	20	0	10.5	630.0	
72	riv. hds	¾"			16.1	11.6	
							954.0
			Lower Laterals.				
12	rods	¾"	21	0	1.5	378.0	
24	Bolts & nuts	½"			22.5	5.6	
6	Hoops				3.	18.0	
							401.6
			Pedestals				
4					100.	400.	400
			Floor System				
114	Planks	2½" x 10"	14	6	4.5	16,530.0	
100	Stringers	3" x 4"	12	0	4.5	5,400.0	
51	Joists	3" x 12"	16	0	4.5	11,200.0	
							33,130.0
1400	Spikes	60 penny			1/10½	133.3	

Summary.

Top Chords	8,612.5
Bottom Chords	2,868.7
Verticals	1,207.0
Diagonals	1,206.4
Bracing	954.0
Lower Laterals	401.6
Pedestals	400.0
Timber	33,130.0
Spikes	133.3

No.	Shape	Length Feet.	In	Wgt. per foot	Total Weight

Total Weight	
of Steel	15783.5 pounds
of Wood	33130.0 pounds
Grand Total	48913.5 pounds
Weight per foot of span	489.1 pounds

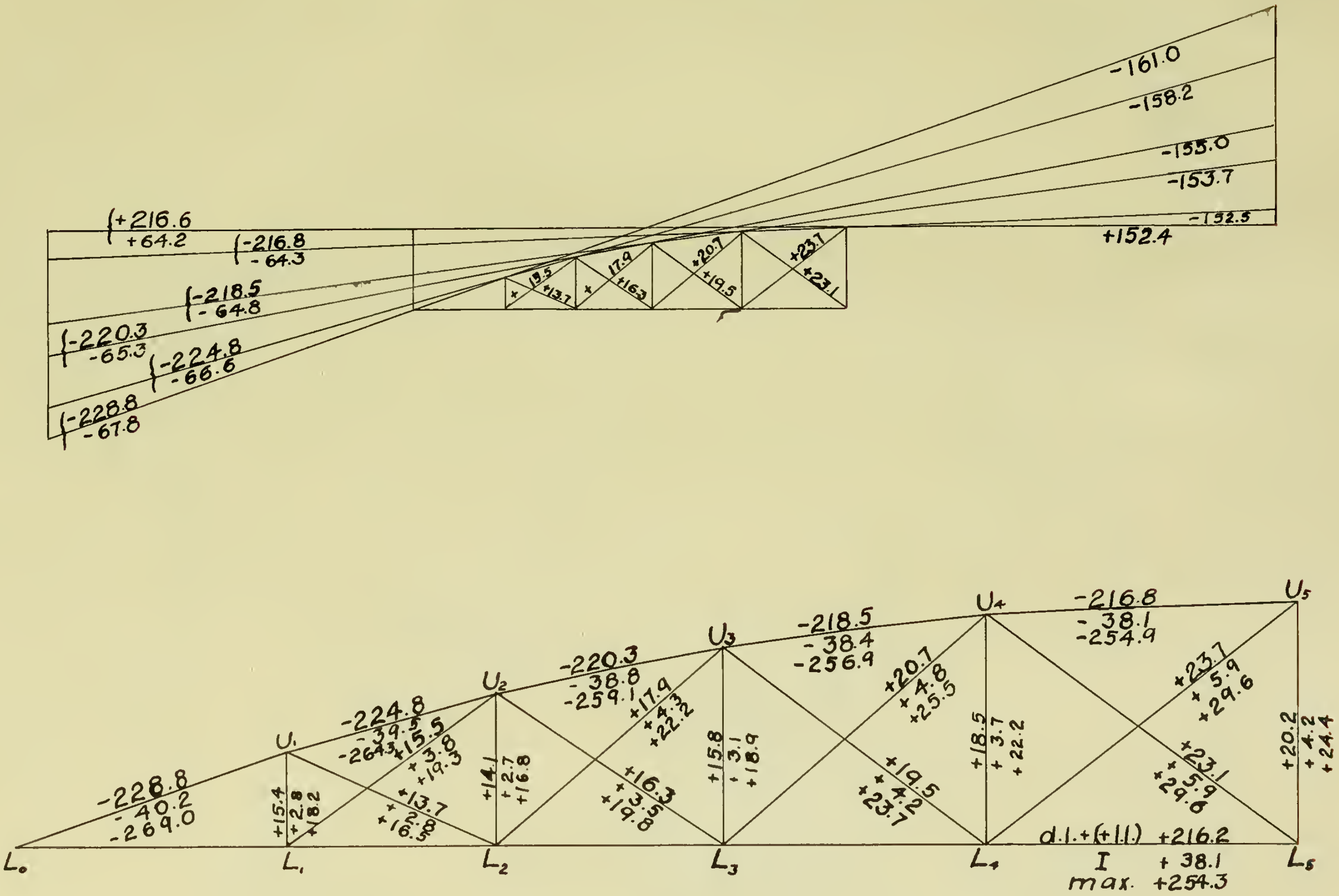


PLATE III GRAPHICAL ANALYSIS of STRESSES in 100-FOOT SPAN

Art. 4. Efficiency of Members.

In the calculation of the efficiency of the members, only the weakest members will be investigated as these are the most important.

LOWER CHORD. The lower chord is made up of two 4" x ½" plates and has a direct stress of 127,000 lbs. For the net area one rivet must be taken out of each plate. The net area equals $2 \times 4 \times \frac{1}{2} - 2 \times \frac{5}{8} \times \frac{1}{2} = 4.0 - .625 = 3.375$ sq. in. The direct unit stress is then equal to $-\frac{127,000}{3,375} = -37,650$ pounds per square inch. Owing to the manner in which the floor beams are connected to the lower chord, secondary stresses are set up in the lower chord. Although the floor beams are spaced two feet apart, the load brought on to the lower chord by them may be considered as a uniform load. This uniform load is made up of the weight of the floor system and the live load. The weight of the floor system is 166.3 lbs. per lineal foot of truss and a live load of eighty pounds per square foot gives 580 pounds per lineal foot of truss, making the total uniform load on the lower chord 746.3 pounds per lineal foot. The loading is shown in Fig. 1. The panel investigated is the middle panel.

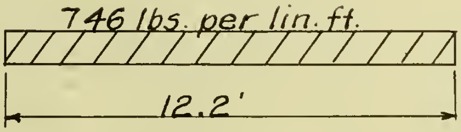


Fig. 1. Loading on Lower Chord Panel.

The unit stress in the extreme fiber of the chords due to this load is given by $S = \frac{Mr}{I + \frac{PL^2}{cE}}$

where $M = \frac{WL^2}{8} = \frac{746 \times 12.2^2 \times 12}{8} = 166,700$ pound-inches.

$Y = 2$ inches

$I = \frac{bd^3}{12} = \frac{64}{12} = 5.33''^3$

$C = 10$

$P = 127,000$ pounds

$E = 29,000,000$ pounds per square inch

$$\text{So } S = + \frac{166,700 \times 2}{5.33 + \frac{127,000 \times 12.2 \times 12^2}{10 \times 29,000,000}} = +21,950 \text{ pounds}$$

The efficiency of the lower chord is then equal to

$$\frac{16,000}{37,650 + 21,950} = 26.85 \text{ per cent.}$$

This shows the extremely unsafe condition of the lower chord. If the bridge had been loaded with a traction engine, this efficiency would be still lower.

Upper Chord. For the upper chord, L_0U_1 will be investigated. The direct stress in this member is -134,500 pounds and the area of the cross section is 11.68 square inches, giving a unit stress of 11,520 pounds per square inch.

The allowable unit load is $16,000 - 70\frac{l}{r} = 16,000 - 70\,\frac{11 \times 12}{2.68} = 12,550$ pounds per square inch. This gives an efficiency of $\frac{12,550}{11,520} = 109$ per cent. The efficiency is probably much less than this since the value of l is taken as the distance L_0U_1, whereas it should be the the unsupported length of the chord, or L_0U_1. Using this value of l gives an allowable unit load of 3,150 pounds per square inch, and an ef-

ficiency of 27.4 per cent. The effect of the wind is neglected because it is somewhat indeterminate, but it would reduce the obtained efficiencies.

Diagonals. As nearly all of the diagonals are the same size, only L_4U_5 will be investigated since it has the highest stress. The stress in L_4U_5 is +14,800 pounds; and it has an area at the root of threads equal to 0.302 square inches, thus giving a unit stress of 49,000 pounds per square inch. The efficiency is $\frac{16,000}{49,000}$ = 32.5 per cent.

Verticals. The vertical having the highest stress is L_5U_5. It has a stress of +12,200 pounds and an area at root of the thread equal to 1.295 square inches, thus giving a unit stress of 9,400 pounds per square inch and an efficiency of one hundred and seventy per cent.

The efficiencies of the several members mentioned above, together with the data necessary for their computation, is given in Table II.

Table II. Efficiency of members.

Member.	Max. Stress lbs.	Area sq. in. Gross	Net	Stress lbs. in.^{2}	All Stress lbs. in.^{2}	Eff. Per cent
Lower chord	+ 127,000	4.00	3.375	37,650		
Secondary Stress				21,950		
Total				59,600	16,000	26.85
Upper chord	− 134,500	11.68	11.68	11,520	12,550	109.0
L_4U_5	+ 14,800	.4418	.302	49,000	16,000	32.5
L_5U_5	+ 12,200	1.767	1.295	9,400	16,000	170.0

Art. 5. Conclusion.

The results of the preceding investigation show that the bridge is very unsafe. As the efficiency obtained for the upper chord is not strictly correct, but should be lower, the verticals are the only members having sufficient section to withstand the stresses. The desirability of a new bridge is obvious; and it is almost beyond one's comprehension that the old structure has been allowed to remain in use.

III. THE NEW BRIDGE.

Art. VI. Specifications.

The specifications to be used in the design of this bridge are Ostrup's Standard Specifications for Highway Bridges, edition of 1911 with the following corrections:

(a) No lower lateral system.

(b) A reinforced concrete floor of the Baker Type, see Fig. 2.

(c) No allowance to be made for impact.

(d) Live load for floor system and its connections a fifteen-ton engine, see Fig.3, or one hundred pounds per square foot of floor surface.

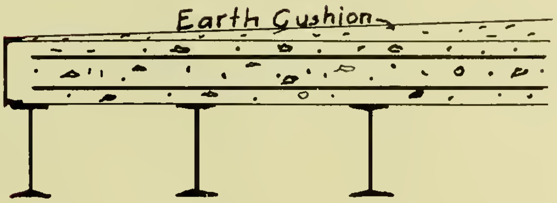


Fig. 2. Baker Floor

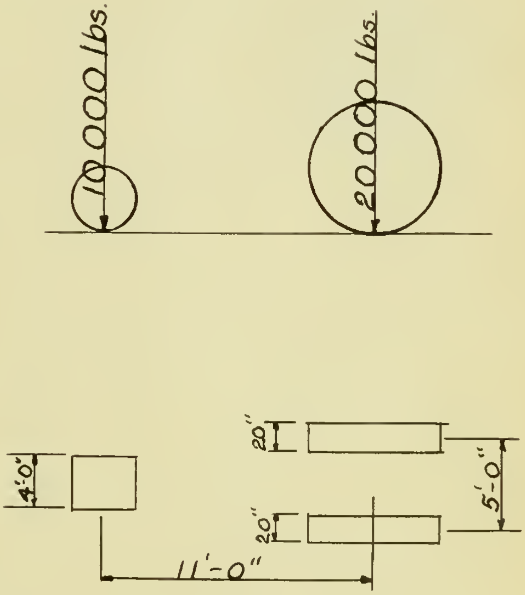


Fig. 3 15-ton Engine

Art. 7. Class.

This bridge is located on a road over which there is considerable traffic; in fact it is the most used road in that part of the country. There is not much chance of the bridge ever being required to carry any electric railroad as there is such a road only one mile north and there is no demand for another electric line over this road. From a consideration of these facts the new bridge will be built as a class "B" highway bridge.

Art.8. Determination of Type.

Before choosing the type of bridge to use, it is first necessary to determine the economical number of spans. The bridge may be built in one span of one hundred and eighty-four feet or in two spans, one of one hundred feet and one of eighty-four feet, thus using the existing pier which is in good shape. The relative cost of these two methods was roughly determined and is presented in Table III. From these figures it is seen that while the masonry for the two spans costs about five hundred dollars more than for the single span, the saving in the total cost is over seventeen hundred dollars, so that the two spans are by far the more economical. These two spans will be made of the through Pratt type, and will have re-inforced concrete floors of the Baker type. The one-hundred-foot span is divided into six panels of 16' -8", and the height of truss is taken as 20'-0". The eighty-four foot span is divided into five panels of 16' -9-9/16"; and the height of truss is taken as 17'-0". These heights are so taken because they are the heights most used for such spans and they leave sufficient clearance. The clear

roadway for both spans is 16'-0" which makes the distance between centers of trusses 17'-0"

Table III

Economical Span Determination.

Material	Unit Price	SPANS 1-100'-0" and 1-84' 0" Quantities	 Total cost	 1-184'-0" Quantities	 Total Cost
Steel	4¢	53,900 lbs.	$2,156.00	112,200 lbs.	$4,488.00
Concrete in pier	$9.50	70 cu. yds.			
in Abutment	9.50	360 cu. yds.	4,080.00	370 cu. yds.	$3,520.00
		430 " "	$ 6,236.00		$8,008.00

Total cost of one span........ $ 8,008.

Total cost of two spans 6,236

Difference in favor of two spans $1,772

For the substructure concrete wing walls and a concrete facing will be added to the existing abutments.

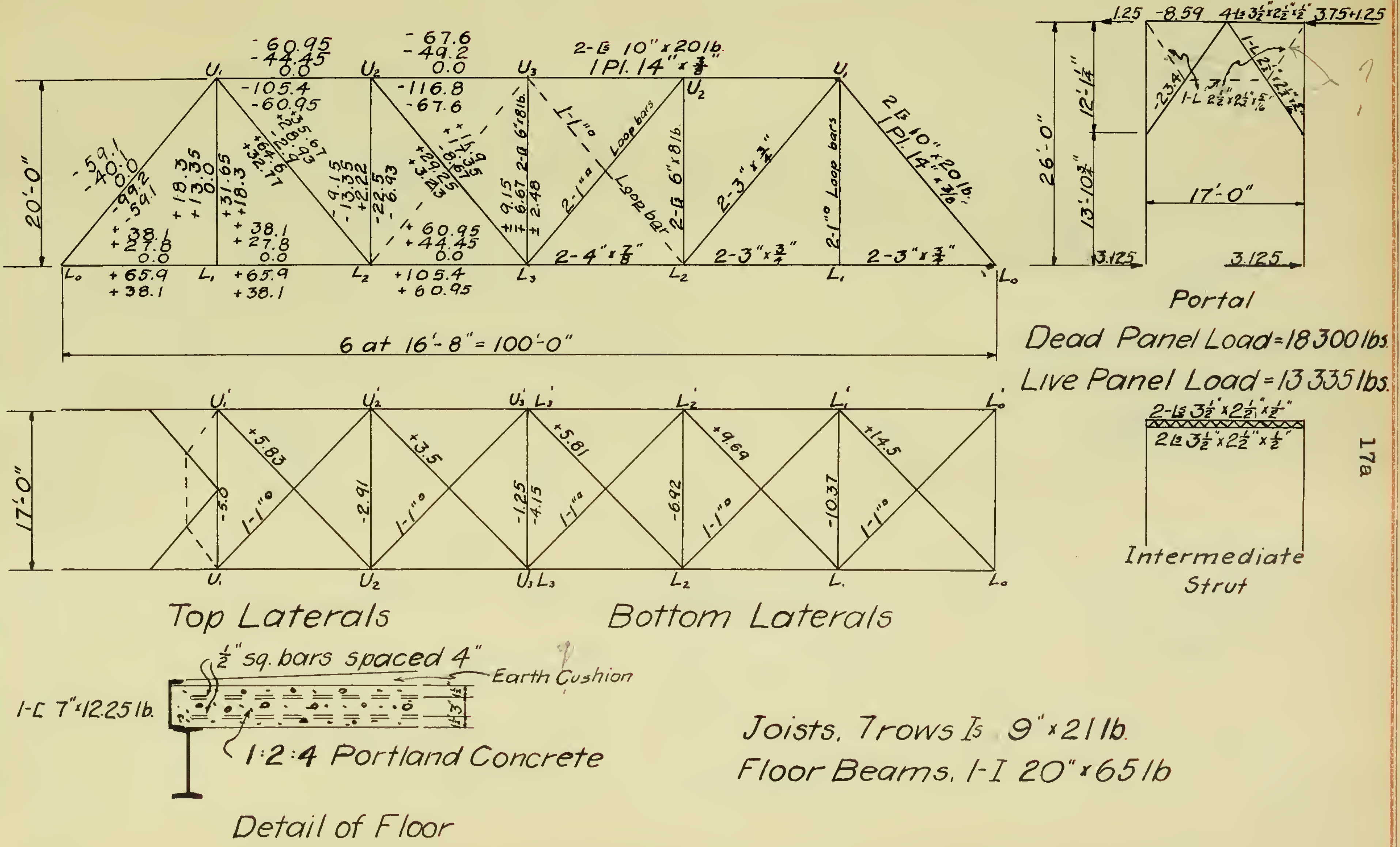
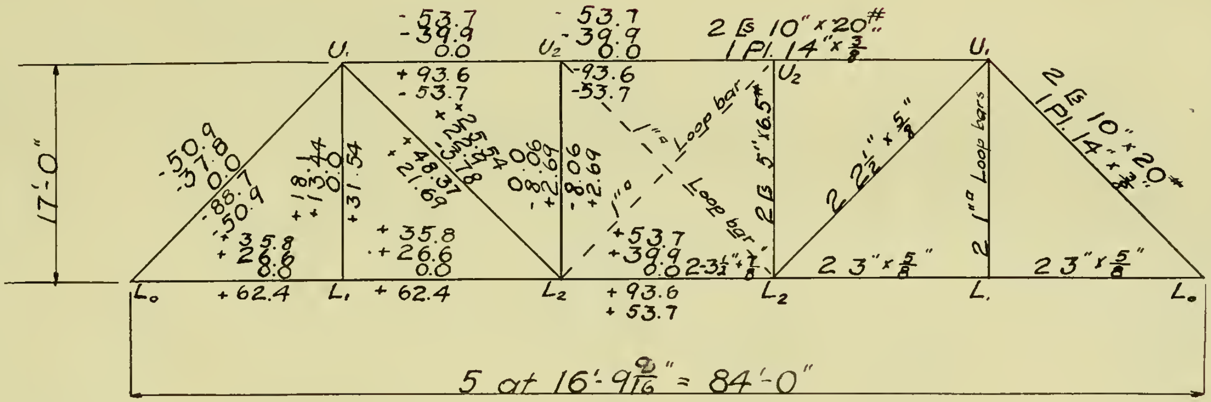
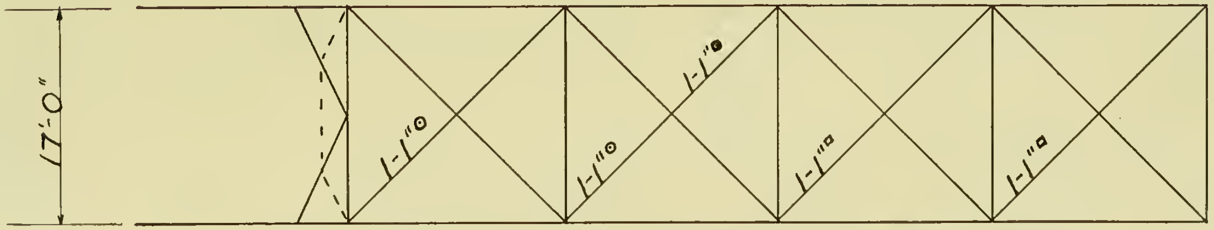


PLATE IV STRESS SHEET of 100-FOOT SPAN



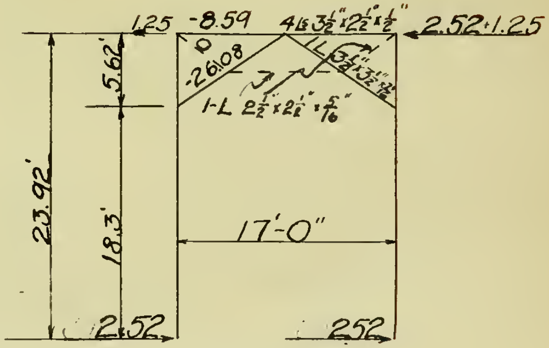


Top Laterals   Bottom Laterals

Floor same as for 100-foot span.

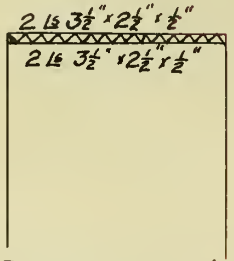
Joists, 7 rows Is 9"×21 lb.

Floor Beams, 1-I 20"×65 lb.



Portal

Dead Panel Load=18100 lbs.
Live Panel Load=13440 lbs.



Intermediate Struts

PLATE V STRESS SHEET of 84-FOOT SPAN

Art. 9. Calculation of Stresses.

For the calculation of the dead load the floor beams are taken as 20" Is x 65# and the stringers as 12" [s x 31.5#. The results are shown in Table IV.

Table IV. Dead Load Weights.

Span. ft.	No. of Panels	Wt. of Steel	Wt. of Floor Beams & Stringers	Wt. of Floors & Cushion	Total weight	Dead Panel Load	Live Panel Load	
100	6	30,800	28,450	160,000	219,250	18,270	26,700	13.35
84	5	23,100	23,660	134,400	181,160	18,116	22,400	13.44

The stresses have been calculated in the usual manner and are shown on Plates IV. and V.

Art. 10. Design of Members.

For the 100-ft. Span.

Floor. In the design of the floor the weight of the heavy wheel of a fifteen-ton engine is considered as uniformly distributed between two stringers. The maximum moment of a uniform load, w, on a continuous slab is taken as $\frac{wl^2}{12}$, so the moment of the wheel is $\frac{10,000 \times 32}{12}$ = 26,680 pounds-inches, thirty two being the distance in inches between stringers. The thickness of the floor is assumed as six inches, thus giving a dead load moment of $\frac{75 \times 32^2}{12}$ = 6,410 pound-inches, making a total moment of 33,090 pounds-inches. The effective depth of the floor is taken as four and one half inches, so R = $\frac{33,090 \times 4}{12 \times 81}$ = 136, giving a value of p = 1.02 per cent, and A = 0.735 square inches. One-half inch square

rods spaced four inches apart in both the top and the bottom of the beam and alternating with each other will be used. Seven-inch, 12.25-pound side channels will be used.

Stringers. The dead load on the stringers consists of the weight of the floor and the weight of the stringer itself. As the stringers are placed 2'-8" centers the weight of the floor per linear foot is 2.67 x 100 pounds = 267 pounds, where one hundred is the weight in pounds of the floor and cushion, seventy-five pounds of which is due to the six-inch floor, and twenty-five pounds due to the three-inch earth cushion. The weight of the stringer will be assumed as 31.5 pounds per foot, thus giving a dead load of 267+ 31.5 = 298.5 pounds per linear foot.

As the fifteen-ton engine of Fig.III does not give as high stresses as the fifteen-ton traction engine of Fig. IV the engine of Fig. 4. will be used in designing the stringers.

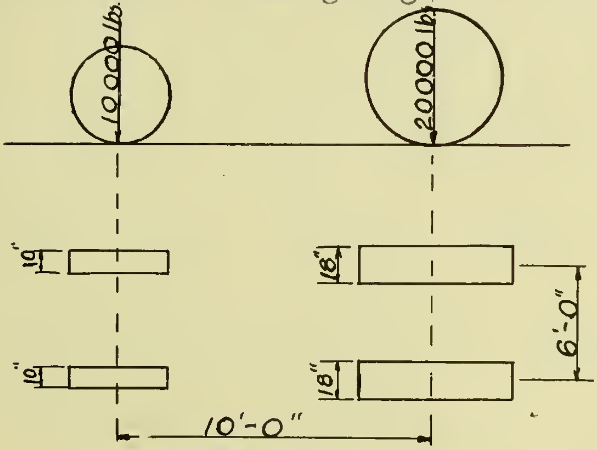


Fig. 4. 15-Ton Engine.

The position of this engine for maximum bending moment is with the heavy wheel in the center of the span. This gives a bending moment = $\frac{1}{3}\left(\frac{10,000}{2}\right)\left(\frac{16.67 \times 12}{2}\right) = 166{,}700$ pound-inches.

The dead load moment is $\frac{298.5 \times 16.67 \times 16.67 \times 12}{8} = 124,300$ pound-inches, and this with the live load moment makes a total of 291,000 pound-inches. This requires a section modulus of $\frac{291,000}{16,000} = 18.2$ inches^3. A nine-inch, twenty-one pound I-beam satisfies this condition and will be used. This weight is less than that assumed but as it will not make a difference of ten per cent in the total stress it is not necessary to re-design.

Floor Beam. The floor beam span is taken equal to the clear roadway, or sixteen feet. As the stringers are spaced so closely together, the dead load will be considered as uniformly distributed over the floor beam. The dead load is,

Floor and cushion 100 x 16 x 16.67 = 26,650 pounds.
Stringers 7 x 21 x 16.67 = 2,450 pounds.
Side channels 2 x 12.25 x 16.67 = 417 pounds.
The dead load on floor beam = 29,517 pounds.

To this must be added the weight of the floor beam itself, which is taken as fifty-five pounds per linear foot, or 16 x 55 = 880 pounds, thus making the total dead load on the floor beam 29,517 + 880 = 30,397 pounds. The dead load bending moment is $\frac{30,397 \times 16 \times 12}{8} = 730,000$ pound-inches.

The greatest bending moment from the engine is caused by placing the wheels as shown in Fig. 5.

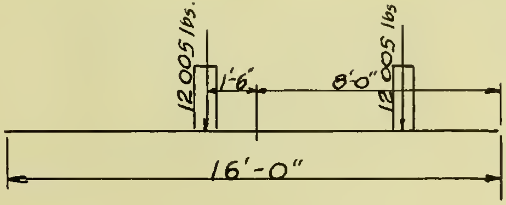


Fig. 5. Loading for Floor Beam.

The small wheel will have $(16.67 - 10)\frac{5000}{16.67} = 2,005$ pounds of its weight transferred to the floor beam, making the loads

12,005 pounds. The left reaction is equal to $12,005\frac{(9.5-3.5)}{16}$ = 9,760 pounds, the moment under the left wheel is 9,760 x 6.5x12= 762,000 pound-inches. The total bending moment is 743,000 + 762,000 = 1,505,000 pound-inches. This requires a section modulus of $\frac{1,505,000}{16,000} = 94.1$ inches^3. A twenty-inch, sixty-five pound I-beam will be used. As this beam has a section modulus of 117.0 it is apparent that a redesign is not necessary.

Tension Members. In the design of the tension members, the width, w, of the member must not be more than $\frac{10}{8}$ of the diameter, D. of the pin, and w should be about six times the thickness, t. The allowable bearing, S, on pins is 24,000 pounds per square inch. Then $SDt = p$, where P is the total stress in the member. Substituting the above values, $t = 0.00295\ P^{\frac{1}{2}}$. The results of the computations are shown in Table V.

Table V. Tension Members.

Member	P Pounds	No of Bars	t inches.	Units Stress Pounds per sq. inch	Area Requir-ed in square inches	Max. Width Inches	Bar Used Inches
	32,900	2	0.535	16,000	2.055	3.21	3 x ¾
	32,900	2	0.535	16,000	2.055	3.21	3 x ¾
	52,700	2	0.677	16,000	3.295	4.06	4 x 7/8
	32,300	2	0.530	16,000	2.020	3.18	3 x ¾
	14,600	2 loop	0.356	16,000	0.912	2.14	1 x 1
		1 loop					1 x 1
	15,800	2 loop	0.371	16,000	0.988	2.23	1 x 1

Intermediate Posts. The least allowable radius of gyration of the vertical posts is $\frac{20 \times 12}{120} = 2.0$.

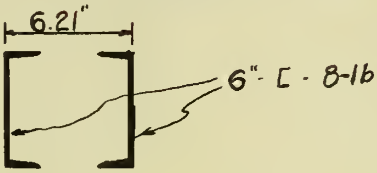


Fig. 6. Intermediate Posts.

A six-inch, eight-pound channel with a radius of gyration equal to 2.34 satisfies this. The allowable unit stress is $16{,}000 - 70\frac{20 \times 12}{2.34} =$ 8,820 pounds per square inch. The required section area is $\frac{22.5}{8820} =$ 2.55 square inches; the area of two of the specified channels is 2 x 2.38 = 4.76 square inches, so these channels will be used. This area is in excess of the required area, but this is the smallest section that satisfies the minimum value of r. To have the radius of gyration equal about both axes the channels must be spaced 6.21 inches apart back to back as shown in Fig. 6. As the stress due to wind is very small it will be neglected.

As the value of $\frac{l}{r}$ governs, the same section must be used for all vertical posts.

Top Chord. The maximum stress in the top chord is 116,500 pounds. A section composed of 2 - 10-inch, twenty-pound channels and a fourteen-inch x 3/8-inch cover plate will be assumed. The distance b-b′ of channels is 14 -2 x 2.74 - 1/8 = 8.4 inches. See Fig. 7. The center of gravity of this section is found to be 6.6 inches above the flange of the channel. The moment of inertia about axis 1-1 is $253.5''^3$, and about axis 2-2 it is $357.3''^3$. The

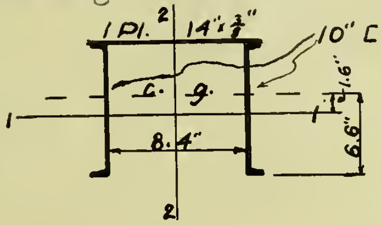


Fig. 7. Top Chord.

minimum value of the radius of gyration is 3.18, making the allowable unit stress $16{,}000 - 70\frac{16.67 \times 12}{3.18} = 11{,}700$ pounds per square inch. The total area of the section is 17.01 square inches, thus making the direct stress equal to -6,850 pounds per square inch. The stress due to eccentricity is equal to

$$-\frac{116{,}500 \times 1.6 \times 6.6}{253.5 - \frac{116{,}500 \times (16.67 \times 12)^2}{10 \times 29{,}000{,}000}}$$

= -4820 pounds per square inch.

The total stress is -6850 - 4820 = -11,670 pounds per square inch so this section is sufficient. This section will be used for the entire top chord.

End Posts. For the end posts the section used for the top chord will be assumed as sufficient and will be investigated. The following stresses were computed.

Direct stress S_1 = 5,830 pounds per square inch.

Stress due to eccentricity S_2 = 4,760 pounds per square inch.

Stress due to weight, S_3 = 1,807 pounds per square inch.

Stress due to wind. S_4 = 11,270 pounds per square inch.

The maximum combination of these stresses will occur as shown in Fig. 8 The allowable stress is 20,000 pounds per square inch, and as the maximum stress exceeds this but very little, the section will be used.

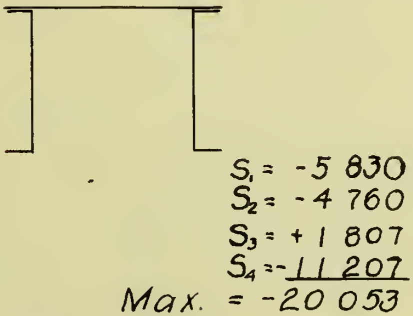


Fig. 8.

Laterals and Cross Bracing. As a concrete floor is used it is not essential that stiff bottom lateral diagonal members be used. The computations for the diagonal laterals are shown in Table VI.

Table VI. Lateral Systems

Member	Maximum Stress in pounds	Area Required Square inches	Section used.
Top Laterals.			
1st. Panel	+5830	0.365	1 rod 1" diameter
2nd. Panel	+35000	0.22	1 rod 1" diameter
Bottom Laterals			
1st. Panel	+14500	0.91	1 rod 1" square
2nd. Panel	+9690	0.61	1 rod 1" square
3rd Panel	+5810	0.36	1 rod 1" square
Portal			
Diagonals	-23.4	2.4	2-∟s 3 x 2½ x 3/8 ⌉⌈

For the intermediate struts four angles latticed together will be used. The minimum allowable radius of gyration is 1.7, so 3½" x 2½" x 5/16" angles with the short legs together and placed 3/8 inches apart will be used. As the total area is over seven square inches it is not necessary to investigate the section.

The same section will be used in the portal strut as in the intermediate struts.

Pins. The pins used are as follows: for U_1 and the lower chord, 3¼ inch pins shall be used, for the upper chord 2¾ inch pins shall be used. These values are taken from tables giving the sizes of pins generally used in highway bridges.

The Eighty-four-foot Span.

For the eighty-four-foot span the method of design will not be given, as this has been fully outlined for the one-hundred-foot-span. The members have been designed and are shown on the stress sheet Plate V.

Art. 11. Recalculation of Dead Load.

The dead load weight of the one-hundred-foot span as designed was calculated. The weight of the steel in the main sections was found to be 47,609 pounds; adding 25 per cent of this, or 11,902 pounds, for details gives a total weight of steel in the one hundred-foot span, equal to 59,511 pounds. The assumed weight of steel as given in Table IV was 59,250 pounds. The weight of the floor and cushion is one hundred pounds per square foot, as assumed. As the actual dead load weight is very close to the assumed weight it is not necessary to re-design.

The assumed dead load weight for the eighty-four-foot span was determined by the same means as for the one-hundred-foot span, so it is reasonable to suppose that it varies only a small amount from the actual dead load weight of the span as designed, so the members will not be redesigned.

Table VII gives the tabulated weight for the one-hundred-foot span.

Table VII. Weights of Members, 100-foot span.

4 End Posts	6,018 pounds

4	Top Chords	7,707
12	Bottom Chords	3,787
12	Diagonals	2,658
4	Hip Verticals	544
6	Intermediate Posts	2,920
5	Floor Beams	5,850
	Stringers	15,050
2	Portals	1,343
	Struts	301
	Bottom Laterals	940
	Top Laterals	491
	Total	47,609
	Plus 25 per cent for details	11,902
	Total	59,511

IV. SUBSTRUCTURE

For the substructure concrete abutments as shown on Plate VI. will be used. An investigation of this abutment gave the following results: the weight, W, of the pier is 264,880 pounds and its center of gravity is 2.63 ft. horizontally from the heel; the maximum load, P, from the bridge is 108,000 pounds, and the center of gravity of W and P is 2.44 ft. from the heel; the maximum pressure due to these forces is $\frac{W + P}{l} + \frac{6(W+P)e}{l^2} = 4360$ pounds per square foot. he minimum pressure is 200 pounds per square foot. As the abutment rests on gravel these values are allowable. Assuming the pressure of the earth as 30 per cent of the pressure due to water, the overturning moment caused by the embankment is found to be 49,500 pound-feet. The resisting moment due to the weight of the pier is 1,220,000 lb.-ft., so the pier is safe.

The part of the old abutment indicated on Plate VI. by dotted lihes, will be used. A 1:2:4 mixture of Portland concrete will be used for work above ground, and 1:3:5 Portland concrete will be used below the surface of the ground. This makes the quantities of concrete to be uaed, as follows: in the pier 12.5 cu. yds. of 1:2:4 concrete and 5.5 cu. yds. of 1:3;6; concrete in the wing walls 23.5 cu. yds of 1:2:4 concrete and 23.5 cu. yds. of 1:3:6 concrete.

The existing pier is in such good shape as not to require any improvements, so it will be used in its present condition.

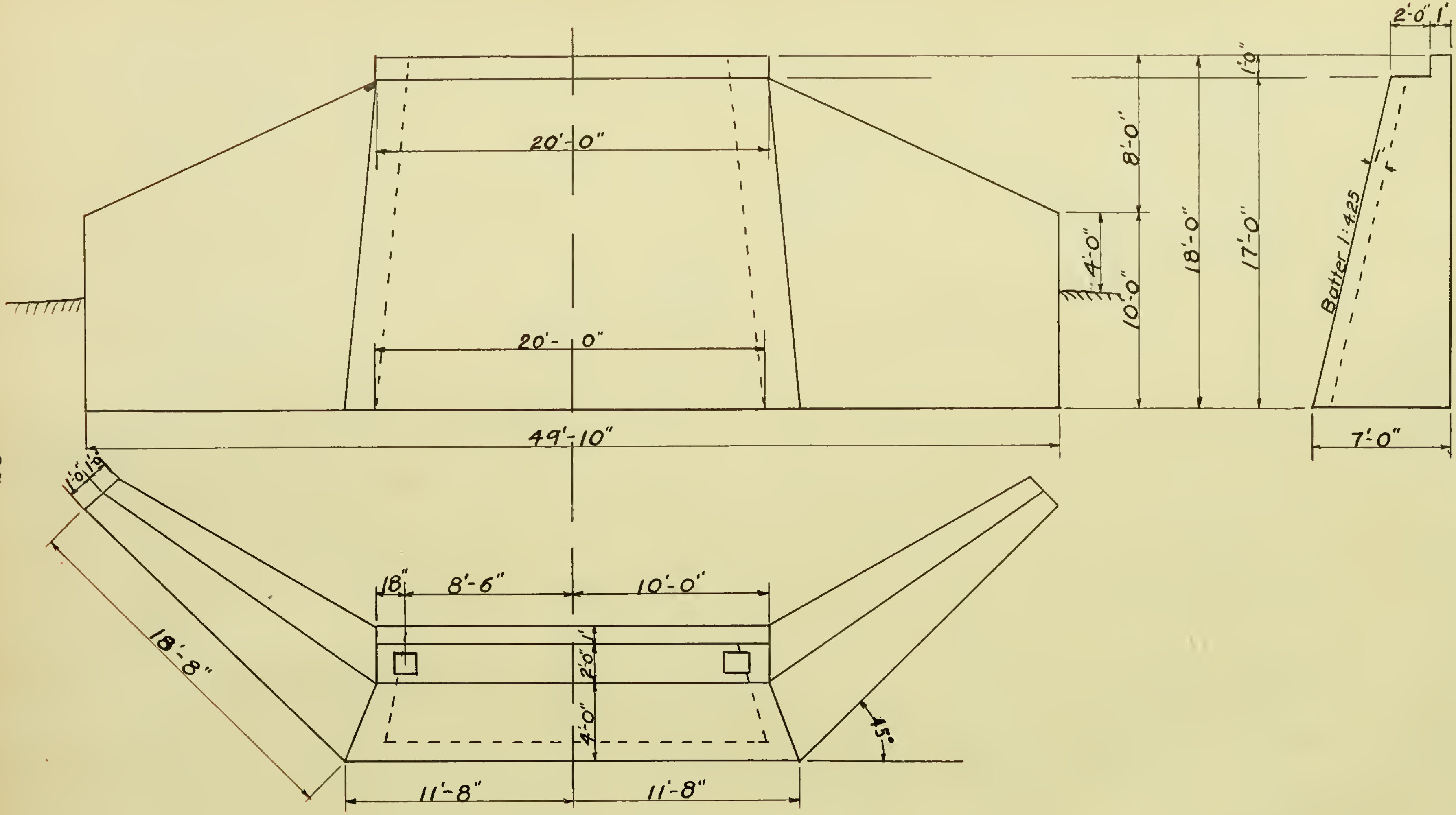


PLATE VI PLAIN CONCRETE ABUTMENT

V. COST.

The cost of removing the old bridge is taken as $1.50 per foot of span, as this was found to be the cost of removing a similar bridge at Mahomet, Illinois, making the total cost of removing the old bridge 184 x $1.50 = $276.00

The cost of the new bridge is presented in tabular form in Table VIII.

Table VIII.

Estimate of Cost of New Bridge.

Metal,	106,010 pounds @ 2.0¢ per pound	$2,120.20
Erection,	184 feet of span @ $2.50 per foot	460.00
Shop,	106,010 pounds @ 0.52¢ per pound	551.25
Haul,	106,010 pounds @ 0.09¢ per pound	95.40
Freight,	106,010 pounds, Pittsburgh to St.Joseph @ 0.22 ¢ per pound	233.22
Incidentals,	106,010 pounds @ 0.20¢ per pound	212.00
Floor,	55.3 cu.yds. 1:2:4 Portland concrete @ $12.50 per cu. yd.	691.25
	15,034 pounds reinforcing rods @ 0.4¢ per pound	601.36
Abutments,	72 cu. yds. 1:2:4 Portland concrete at $9.50 per cu. yd.	684.00
	58 cu. yds. 1:3:6 Portland concrete at $8.00 per cu. yd.	464.00
	First total	6,122.08
	Profit, 15 per cent of first cost	918.40
	Total cost	$7,041.08

Cost of removing old bridge	$ 270.00
Final cost	$ 7,317.08

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